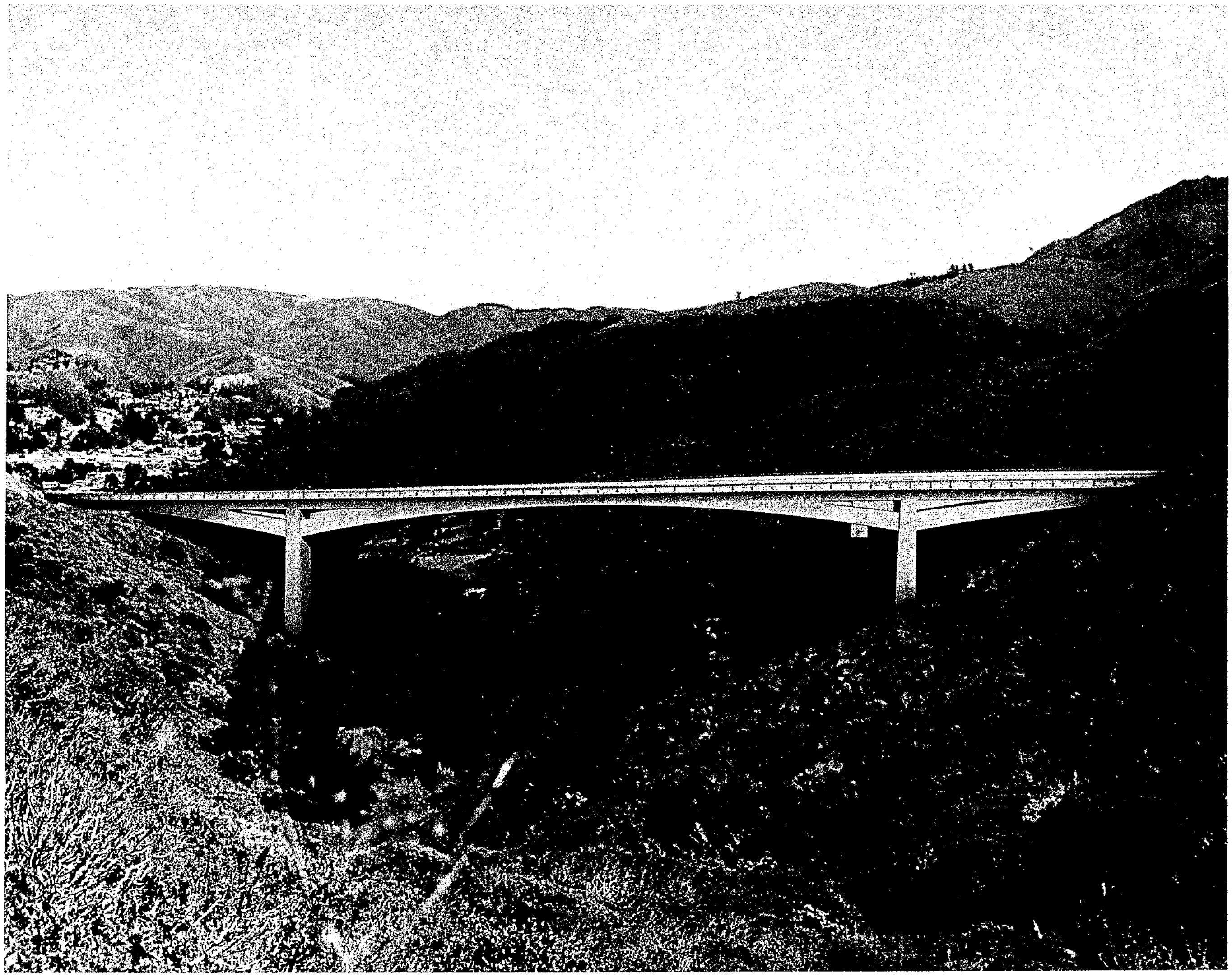
Devil's Slide Bridge

Br. No. 35-0331R/L **EA: 04-1123K1**

DESIGN CRITERIA

Version 2.11 5/5/04



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5. ANALYSIS AND DESIGN

1. GENERAL

This Design Criteria is for cast-in-place prestressed segmental concrete box girder bridges constructed by the balanced cantilever method.

The bridges shall be designed in accordance to the Caltrans Bridge Design Specifications (BDS) LFD Version, April 2000 (1996 AASHTO with interim's and revisions by Caltrans).

In addition to the Bridge Design Specifications (BDS), pertinent sections of the following codes or criteria's are to be used unless otherwise noted or revised in this criteria.

AASHTO Guide Specifications For Design and Construction of Segmental Concrete Bridges. Second Edition 1999 with 2003 interims (AASHTO Seg).

CALTRANS Seismic Design Criteria Version 1.2, December 2001 (SDC)

CALTRANS Memo to Designers (MTD)

AASHTO LRFD Bridge Design Specifications, 1998 with Interims up to 2002 (used for shear and torsion design only)

1.1. Superstructure Definition

The superstructure of the bridge refers to both the "box girder" and the "strut". The "box girder" refers to the portion of the structure that includes the deck surface that the traffic rides on. The "strut" refers to the lower unit of the structure after the "box girder" has split vertically into two members near the piers.

2. DESIGN LOADS

2.1. Structural Dead Loads

The Dead Load of the structure shall be based on a unit weight of concrete (including rebar) of 2483 kg/m^3 (155 lb/ft^3). (AASHTO Seg 7.4.1)

The deck of the bridge shall include an integral 25 mm (1inch) overlay to provide for profile grinding of the bridge deck. This 25 mm (1 inch) additional deck thickness for the integral-wearing surface shall be added to the required 50 mm (2 inch) cover of the deck reinforcement.

2.2. Construction Loads

The bridge shall be designed for construction erection dead and live loads and combinations based on the AASHTO Segmental Specification Section 7.4.

The assumed form traveler loading (CE) for the contract plans analysis shall be as follows:

Traveler weight	534 kN (120 kips)
Formwork	223 kN (50 kips)
Center of gravity	1.0 m (3.3 ft) in front of leading edge of supporting segment

The Contractor shall base his analysis on the actual form traveler equipment to be used on the job.

2.3. Superimposed Dead Load

Permanent loads applied to the structure after completion of the segmental construction (SDL) shall include the following:

Future wearing surface	1.675 kPa (35 psf)
Modified Type 80 Concrete Barrier	580 kg/m (390 lb/ft)
Tubular Bicycle Railing	60 kg/m (40 lb/ft)
Hoist Trolley Rail	34 kg/m (23 lb/ft)
Utilities (water supply line and/or electrical and communication)	370 kg/m (250 lb/ft)

2.4. Live Loads (LL + I)

HS20-44 and alternative and permit design load per BDS Specification.

The design speed (S) for calculating the centrifugal forces shall be as follows:

HS Truck:	S = 80 km/hr (50 mph)
Permit Truck:	S = 40 km/hr (25 mph)

2.5. Thermal Effects

The Bridge is located in the mild California State Highway Environmental Area 1 (See Memo To Designers 8-2).

2.5.1. Ambient Temperature Range

The Ambient temperature range is based on record high and low temperatures for the city of Pacifica (www.weather.com).

Record High Temperature:	41°C (106°F)
Record Low Temperature:	4° C (24°F)

The Normal Mean Temperature for the area is 14°C (58°F).

2.5.2. Design Temperature Range for Thermal Forces

The design temperature range for calculating forces in the structure (TRF) shall be 17°C (30°F) (rise or fall). The temperature range above accounts for the lag between the air temperature and the interior of massive concrete members or structures. (BDS 3.16)

The movement per unit length = .00018 (Coefficient of thermal expansion = .0000108/°C (0.0000060/°F)

2.5.3. Temperature Gradient – TG

The positive and negative thermal gradients shall be applied per AASHTO Segmental Guide Specifications 6.4.4. The positive and negative gradients shall correspond to the values below.

Positive Gradient	Negative Gradient
T1 = 54°F (30°C)	T1 = -16.2°F (-9.0°C)
T2 = 14°F (7.8°C)	T2 = -4.2°F (-2.3°C)
T3 = 0°F (0°C)	T3 = 0°F (0°C)
A = 12" (300mm)	A = 12" (300mm)

The temperature gradient shall not be applied to the strut member.

2.6. Creep and Shrinkage

The effects due to creep and shrinkage (C+S) shall be accounted for in the design as specified in the 1990 CEB_FIP Model Code.

The shrinkage coefficient shall be calculated based on an average relative humidity for the site of 70% and an "Average Ambient Temperature" value shown in Section 2.5. Portland Cement shall be assumed to be Type II Modified with a medium water/cement ratio.

2.7. Seismic

2.7.1. Seismic Loading during Construction

During the segmental construction, the bridge shall be designed to resist forces from a probabilistic ground motion with a 10% probability of exceedence in 2 years (19 year return period). Seismic forces may be determined using the cracked flexural and torsional stiffness of the piers (effective moment of inertia). See Figure 5.1.1 for the acceleration response spectra curve with 5% damping.

The loading combination shall be the Group VII load combination in Table 2.8.1.

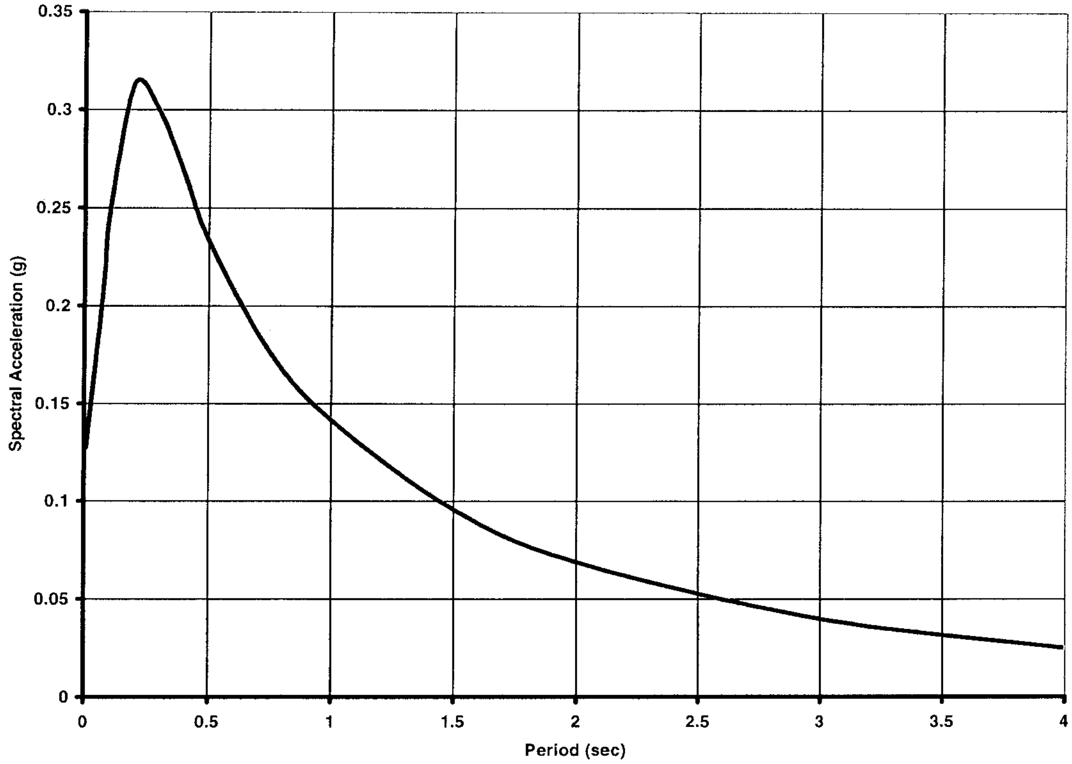

FIGURE 5.1.1
EQUAL HAZARD ACCELERATION RESPONSE SPECTRA CURVE
(2 Year Life Span with 10% Probability of Exceedence)

2.7.2. Seismic Loading on Final Structure

The design loading on the completed structure shall be based on a five-percent damped site-specific elastic Acceleration Response Spectrum (ARS), which is the Maximum Credible Earthquake (MCE) for the site. See the "General Notes" in the "Bridge Plans" for the ARS curve.

The effects of a vertical acceleration on the superstructure shall be approximated by the equivalent static load method of the Caltrans SDC Section 7.2.2 except as noted. At the end spans, the bridge shall be assumed to be roller supported for downward loads and a cantilever for upward loads. The Designer may include the prestress tendons in determining the superstructure capacity with the following conditions:

1) The uniformly applied vertical force shall be 25% of the dead load applied upward and 25% of the dead load applied downward.
2) The vertical force shall be added to the Dead Load, Prestressing, Creep and Shrinkage forces.
3) The Capacity shall be based on nominal material properties and a phi factor for flexure 0.95.

2.8. Combination of Loads

2.8.1. Construction Load Combinations

Service level segmental construction loads shall be investigated for load combinations "a" through "f" in AASHTO Guide Specification Section 7.4 and Table 7-2.

The Construction load combinations for the Load Factor Design check of the post-tensioned superstructure shall be:

1) 1.1(DL + DIFF) + 1.3(CE + CLL) + 1.0(PS + C+S)

2) Group VII load combination in Table 2.8.1.

The construction load combinations for the Load Factor Design of the reinforced concrete struts and piers shall be as shown in Table 2.8.1.

TABLE 2.8.1 Factors for Load Factor Design (Construction Loads)

		Beta Factors							
Group	Gamma Factor	D + DIFF + CE	CLL	W + WUP	PS	C+S	TRF	TG	EQ
I	1.30	β_D	1.67	0	0.77	0.77	0	0	0
II	1.30	β_D	0	1	0.77	0.77	0	0	0
III	1.30	β_D	1	0.3	0.77	0.77	0	0	0
IV	1.30	β_D	1	0	0.77	0.77	1	0	0
V	1.25	β_D	0	1	0.80	0.80	1	0	0
VI	1.25	β_D	1	0.3	0.80	0.80	1	0	0
VII	1.00	1	0	0	1.00	1.00	0	0	1

$\beta_D = 0.75$ when checking piers for maximum moment or maximum eccentricities and associated axial load; and when dead load effects are of opposite sign to the net effects of other loads in a group.

$\beta_D = 1.00$ when checking piers for maximum axial load and associated moment

$\beta_D = 1.00$ when checking struts

D	Dead load
DIFF	2% differential dead load on one cantilever
CE	Specialized construction equipment (form traveler)
CLL	Construction live load
W	Wind load on structure
WUP	Wind Uplift on one cantilever
PS	Prestress
C	Creep
S	Shrinkage
TRF	Temperature (Thermal - Rise or Fall)
TG	Temperature Gradient (Thermal – Differential)
EQ	Earthquake during construction (see Sect 5.1.1)

2.8.2. Final Load Combinations

All other Final Loading Combinations after completion of the segmental construction and closures shall be based on Caltrans BDS 3.22 and as shown in Tables 2.8.2A and 2.8.2B with consideration of the following additional loadings (*AASHTO Seg 7.2*):

1) The permanent effects of creep (C) and shrinkage (S) shall be added to all loading groups with a load factor of 1.0 (gamma * beta = 1.0).

   Creep and Shrinkage effects (C&S) shall be evaluated for all load combinations on both a young and an old structure. The "young structure" shall be at the time when all closures have been made. The "old structure" shall be a minimum of 30 years old.

2) The final state erection loads (EL), defined as the final accumulated "built-in" forces and moments resulting from the construction process, shall be included in all loading combinations. The term "D" in BDS 3.22 shall be redefined as:

   D = (DL + SDL + EL)

   Where:

   DL = Dead load of structure only

   SDL = Additional superimposed dead load (See Section 2.3)

   EL = Erection Loads (final state)

TABLE 2.8.2A Factors for Service Load Design (Final Structure)

		Beta Factors										
Group	Gamma Factor	D	L+I	CF	W	WL	LF	PS	C+S	TRF	TG	% Allow
I	1.00	1	1	1	0	0	0	1	1	0	0	100
II	1.00	1	0	0	1	0	0	1	1	0	0	125
III	1.00	1	1	1	0.3	1	1	1	1	0	0	125
IV	1.00	1	1	1	0	0	0	1	1	1	0.5	125
V	1.00	1	0	0	1	0	0	1	1	1	1	140
VI	1.00	1	1	1	0.3	1	1	1	1	1	0.5	140
X	1.00	1	0	0	0	0	0	1	1	0	1	100

TABLE 2.8.2B Factors for Load Factor Design (Final Structure)

		Beta Factors										
Group	Gamma Factor	D	(L+I) H	(L+I) P	CF	W	WL	LF	PS	C+S	TRF	TG
I_H	1.30	β_D	1.67	0	1	0	0	0	0.77	0.77	0	0
I_{PC}	1.30	β_D	0	1	1	0	0	0	0.77	0.77	0	0
I_{PW}	1.30	β_D	1	1.15	1	0	0	0	0.77	0.77	0	0
II	1.30	β_D	0	0	0	1	0	0	0.77	0.77	0	0
III	1.30	β_D	1	0	1	0.3	1	1	0.77	0.77	0	0
IV	1.30	β_D	1	0	1	0	0	0	0.77	0.77	1	0
V	1.25	β_D	0	0	0	1	0	0	0.80	0.80	1	0
VI	1.25	β_D	1	0	1	0.3	1	1	0.80	0.80	1	0

H denotes H live loads
PC denotes Permit live loads used only on superstructure (box girder and strut).
PW denotes Permit live load used on substructure.
$\beta_D = 0.75$ when checking piers for maximum moment or maximum eccentricities and associated axial load; and when dead load effects are of opposite sign to the net effects of other loads in a group.

$\beta_D = 1.00$ when checking piers for maximum axial load and associated moment

$\beta_D = 1.00$ when checking struts

D	Dead load
L	Live load
I	Live load impact
W	Wind load on structure
WL	Wind on live load – 100 pounds per linear foot
LF	Longitudinal force from live load
CF	Centrifugal force from live load
PS	Prestress
C	Creep
S	Shrinkage
TRF	Temperature (Thermal - Rise or Fall)
TG	Temperature Gradient (Thermal – Differential)

3. ALLOWABLE STRESSES – Superstructure

3.1. Transverse Box Girder Design

The following allowable stresses shall be used for the prestressed concrete deck design:

1) Temporary Stresses Before Losses Due To Creep and Shrinkage.

Compression: $0.55f'_{ci}$

2) Final Stresses after Losses Have Occurred.

Compression: $0.4\ f'_c$

Tension at Neg. Moment regions (top of deck): 0

Tension at Pos. Moment regions (bottom of deck): $0.25\sqrt{f_c'}$ MPa ($3\sqrt{f_c'}$ psi)

The allowable stress for the prestressing steel shall be as shown in Section 3.2.1

3.2. Longitudinal Superstructure Design

Stress analysis shall include the effects of the prestress duct voids prior to grouting and the prestress tendon area in the transformed section analysis.

The following allowable stresses are for the longitudinal Service Level Design of the post-tensioned superstructure. For exceptions at the strut member, see Section 5.1.

3.2.1. Prestressing Steel

3.2.1.1. Maximum Jacking Stress (BDS 9.15.1): $0.75f'_s$

3.2.1.2. Stress Immediately after Seating at Anchorage: $0.75f'_s$

3.2.2. Prestressed Concrete

Allowable stresses per BDS Specification 9.15 unless otherwise noted. For allowable stresses of the transverse prestress design of the deck see Section 3.1 above.

3.2.2.1. Temporary Stresses Before Losses Due to Creep and Shrinkage

3.2.2.1.1. Compression: $0.6 f'_{ci}$

3.2.2.2. Stresses at Service Load After Losses Have Occurred

For Definition of loading abbreviations, see Section 2.8.2 and notes at bottom of Table 2.8.2B

3.2.2.2.1. Tension in the precompressed tensile zone

3.2.2.2.1.1. Segmental Construction Loading Cases "a" through "f": See AASHTO Seg. Table 7-2 [varies $0.5\sqrt{f_c'}$ to $0.58\sqrt{f_c'}$ MPa, ($6\sqrt{f_c'}$ to $7\sqrt{f_c'}$ psi)].

3.2.2.2.1.2. Full Dead Load, D : 0

3.2.2.2.1.3. Service Load Design Group I-X (see Table 2.8.2A): $0.5\sqrt{f_c'}$ MPa ($6\sqrt{f_c'}$ psi)

3.2.2.2.2. **Compression**

3.2.2.2.2.1. Segmental Construction Loading Cases "a" through "f" (See AASHTO Seg. 7.4.2): $0.5 f'_c$

3.2.2.2.2.2. ½ D + L + ½ PS + ½ (C+S): $0.4 f'_c$

3.2.2.2.2.3. Full Dead Load (D) + effective PS: $0.4 f'_c$

3.2.2.2.2.4. Service Load Design Group I-X (Table 2.8.2A): $0.6 f'_c$
(Except as noted above)

(Stress reduction required if flange or web slenderness ratio > 15: see AASHTO Seg. 9.2.2.1)

4. MATERIALS

4.1. Concrete

4.1.1. Superstructure (box girder & struts) And Piers

f'_c = 6100 psi (42 MPa)

4.1.2. Other

f'_c = 3600 psi (25 MPa)

4.2. Prestressing Steel

4.2.1. Ultimate Stress, f'_s: 1860 MPa (270 ksi) low relaxation strand

4.2.2. Assumed Anchor Set: 10 mm (3/8 inch) longitudinal & transverse

4.2.3. Friction Losses (BDS 9.16.1).

Friction Coefficient, μ: 0.15

Wobble Coefficient, K: 0.000656 rad/m (0002 rad/ft). Factor used in AASHTO friction eqn.

Wobble Coefficient, B: 0.2506 deg/m (0.07639 deg/ft) Factor used in European friction eqn.

5. ANALYSIS AND DESIGN

5.1. General

Due to the horizontal curvature of the bridge, all load cases shall be analyzed using three-dimensional structural analysis software. The time dependent construction analysis by the Contractor shall also incorporate three-dimensional analysis software.

The software that was used for the global design of the bridge is as follows:

TDV RM2000 Spaceframe:	Used for all loads including time dependent analysis and seismic elastic dynamic analysis (EDA).
SAP2000:	Used for elastic dynamic analysis (EDA), Inelastic static analysis (ISA), and non-linear time history dynamic analysis.

5.2. Box Girder And Strut

For the post tensioned elements of the superstructure (this includes both the longitudinal and the transverse design of the box girder) the prestressing force and the required concrete strength shall be determined by the Service Load Design Method (Allowable Stress Design) using elastic theory for loads at the service level considering HS live loads (BDS 9.13.1.2)

The ultimate moment capacity of the post-tensioned members (box girder) shall be based on the Strength Design Method (Load Factor Design) with factored HS and/or P live loads.

The strut shall be considered a partially prestressed element, when checking allowable stresses, with the following requirements:

1) For construction load combinations and final dead load only, the strut shall be designed to meet the allowable tension and compression stress requirements of the post tensioned superstructure.
2) For all other load combinations the strut shall be designed as follows:
    a) Allowable compression stresses shall be per the requirement of prestressed elements (post-tensioned superstructure requirements above).
    b) Strut does not have to meet any Service Load Design tension limits.
    c) The strut shall be designed as a reinforced concrete compression member by the Strength Design Method and reinforced adequately for any tension requirements. The strut shall meet the requirements of both concrete arches in BDS 8.14.3 and hollow rectangular compression members in BDS 8.16.4.4. The Strut shall be reinforced per the requirements for hollow rectangular compression members in BDS 8.17.4.

5.3. Transverse Box Girder

The transverse design of the box girder for flexure shall consider the sections as rigid box frames. Top slabs shall be analyzed as variable depth sections considering the fillets between the top slab and webs (BDS 9.7.3.2 & AASHTO Seg 4.0). BDS 3.24.3 shall not apply.

The bridge deck shall be transversely prestressed to provide a higher level of service consistent with the longer life expectancy of major segmental bridges. The deck shall be designed by the

Service Load Design Method (Allowable Stress Design) using HS live loading and checked for ultimate moment capacity by the Strength Design Method (Load Factor Design) using factored HS and/or P live loads.

The soffit slab shall be designed to support a prestress tensioning jack transported along the spans on a dolly. The dead load of the tensioning jack shall be assumed to be a 22 kN (5.0 kip) concentrated point load. The design shall include a 30% impact factor. This load shall be applied in combination with the $(L+I)_H$ in Tables 2.8.2A and 2.8.2B (Live load secondary effects with live load on the deck).

The deck slab shall be designed for shear according to BDS 9.20. The soffit slab shall be designed for shear according to BDS 8.16.6.

5.4. Shear And Torsion

The shear and torsion design for the superstructure (including the strut) shall be per the AASHTO LRFD Bridge Design Specifications, 1998 with interims up to 2002.

The combined shear flows from shear and torsion shall be considered per AASHTO LRFD Bridge Design Specifications 5.8.3.6 or AASHTO Segmental Guide Specification 12.2.

The methods for design of web reinforcement in the Bridge Design Specifications (BDS) and the AASHTO Guide Specifications (AASHTO Seg) are an acceptable alternate.

5.5. Girder Stirrups

The stirrups in the girder webs shall be designed for the longitudinal shear and torsion (A_v) and the transverse bending from the transverse box girder analysis (A_f). The minimum area of steel should not be less than the larger of the following combinations of the two effects:

a) $A_v + 0.5A_f$

or b) $0.5A_v + A_f$

or c) $0.7(A_v + A_f)$

(Construction and Design of Prestressed Concrete Segmental Bridges, Podolny & Muller, page 203)

The stirrups shall also be designed for the bending moment in the girder web (A_b) due to the lateral prestress force of longitudinal tendons that are located within the girder webs along the curved portions of the bridge. The stirrup leg area shall be the larger of A_b or the controlling (A_v + A_f) case above. The design (A_b) shall be according to MTD 11-31.

5.6. Seismic

Earthquake horizontal effects shall be determined from horizontal ground motions applied by Method 1 or Method 2 according to SDC 2.1.2.

5.6.1. During Construction

Seismic design forces shall be less than the design strength of the various structure components. The design strength shall be determined per the Bridge Design Specifications (BDS) 8.16, with the exception that the strength reduction factor (phi) for flexure in Group VII columns shall be 1.0 (BDS 8.16.1.2.2).

5.6.2. Final Structure

The bridge shall be designed to prevent collapse during the Maximum Credible Earthquake (MCE) and meet all the provisions of the Caltrans Seismic Design Criteria (SDC) except as noted in these Criteria.

The bridge shall be designed to meet displacement ductility requirements of the Caltrans Seismic Design Criteria (SDC) using both a three-dimensional elastic dynamic analysis (EDA) and an inelastic static analysis (ISA).

The superstructure shall be designed to resist the internal forces generated when the structure has reached its Collapse Limit State in the longitudinal direction (see SDC 2.3 and "Memo To Designers" 20-6). The secondary prestress moment in the strut shall be included in the analysis regardless of weather the overall demand is increased or decreased.

The superstructure shall also be designed to resist internal forces generated from transverse displacement using a three-dimensional inelastic static analysis (ISA). Because of the curved alignment, transverse pier displacements will induce flexure, shear, and torsional forces into the superstructure. The superstructure shall be designed to resist the internal forces developed when the pier has displaced an amount equal to the predicted transverse deflection from the elastic dynamic analysis (EDA).

The bridge shall be checked using a three-dimensional, non-linear time history dynamic analysis to verify that the simplified design procedures presented above were adequate to capture the behavior of the structure

5.7. Piers

The reinforced concrete piers and footings shall be designed by the Strength Design Method for both construction and final load combinations according to BDS Section 8. In addition to the usual substructure design considerations, unbalanced cantilever moments due to segment weights and erection loads shall be accommodated in the pier design (BDS 9.7.3.1.3).

If temperature and shortening loads control the design of the pier, then the temperature and shortening forces should be recalculated using the effective moment of inertia of the piers (SDC 5.7).

The displacement capacity of the pier shall be determined according to SDC Section 3 and meet the demand versus capacity requirements of SDC Section 4.

The footing flexural and shear capacity (reinforced concrete) shall be designed to resist the overstrength moment capacity ($Mo = 1.2 * Mp$) of the pier (SDC 4.3 & 7.7). The footing shall be proportioned to meet the joint shear requirements of SDC 7.7.1.4

The pile analysis shall be based on resisting the plastic moment capacity (Mp) of the pier using a conventional elastic analysis assuming that the piles are axial load members only (competent soil with simplified model per SDC 7.7.1.1).

5.8. Abutments And Retaining Walls

Abutments and retaining walls shall be designed by the Service Load Design method (BDS 3.20, 5.0, and 8.0).

5.9. Bearings

The steel reinforced elastomeric bearing pads at the abutments shall be designed according to Memo To Designers 7.1 with the following modifications:

1) The "Prestress Shortening" shall be the shortening due to creep and shrinkage effects (C+S) determined according to Section 2.6 for both the young and old structure as defined in Section 2.8.2.
2) The minimum bearing pad thickness for the sliding bearing (elastomeric bearing with greased plate) shall be 2[1.5(temperature movement) + 0.25(C+S shortening)]
3) Thermal effects (movement and reactions) on the bearings shall include temperature rise or fall (TRF in section 2.5.2) and the temperature gradient (TG in section 2.5.3). The combination of Live load, TRF, and TG shall be according to Table 2.8.2A.
4) The maximum thickness for steel reinforced bearing pads shall be 200 mm (8 inch)
5) The Shear Modulus (adjusted) shall be based on the record low ambient temperature shown in Section 2.5.1
6) For steel reinforced bearing pads with a Shape Factor greater then 7.5, the average pressure shall not exceed 8.3 MPa (1200 psi).

5.10. Expansion Joint Assemblies

The expansion joint assemblies shall be designed according to Memo To Designer 7.10 with the following modifications:

1) The "Anticipated Shortening" shall be the shortening due to creep and shrinkage effects (C+S) that are anticipated to occur between the casting of the abutment diaphragm and 30 years in the future determined according to Section 2.6.
2) The anticipated shortening shall be increased by 25% (when the MR is greater than 100 mm).